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An experimental investigation on the seismic behavior of cold-formed steel walls sheathed by thin steel plates

Mohammad Reza Javaheri-Tafti [a], Hamid R. Ronagh [b,*], Farhad Behnamfar [c], Parham Memarzadeh [d]

[a] Civil Engineering Department, Isfahan Science and Research Branch, Islamic Azad University, Isfahan, Iran
[b] School of Civil Engineering, the University of Queensland, Brisbane, Qld. 4072, Australia
[c] School of Civil Engineering, Isfahan University of Technology, Isfahan, Iran
[d] Civil Engineering Department, Najafabad Branch, Islamic Azad University, Isfahan, Iran

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ABSTRACT

The use of cold-formed steel (CFS) frames has grown extensively in recent years, particularly in the earthquake-prone regions. However, the behavior of lateral resisting systems in CFS structures under seismic loads has not been scrutinized in detail. Towards this, an experimental investigation has been conducted on cold formed steel frames sheathed by thin galvanized steel plates, the results of which are presented here. The experiments involve 24 full-scale steel plated walls tested under cyclic loading with different configurations of studs and screws. Of particular interest were the specimens' maximum lateral load capacity and the load-deformation behavior as well as a rational estimation of the seismic response modification factor, *R*. The study also evaluates the failure modes of the systems. The main factors contributing to the ductile response of these shear walls are also discussed in order to suggest improvements so that the walls respond plastically with a significant drift and without any risk of brittle failure.

1. Introduction

Lateral force resisting systems in light-framed cold-formed steel (CFS) structures commonly consist of CFS framed walls combined with structural plywood, oriental strand board or thin steel plates to form a shear wall. Thin steel plating is sometimes the preferred option particularly due to its esthetic consistency with the rest of the frame, the higher achievable capacities and the perceived higher ductility. Fig. 1 shows a typical CFS shear wall with sheathing. The sheathing is fastened to the frame around the boundary elements and onto the interior studs by self-drilling screws. As the highest forces are developed at the boundary, screw spacing is much smaller at the boundary (perimeter) of the steel plate than in the middle. Hold-downs often in the shape of bolted angles are installed on the boundary studs to resist the overturning forces developed by the lateral forces.

In order to design lateral bracing, one can refer to the North American Standard for Cold-Formed Steel Framing – Lateral Design (AISI S213) [1] which provides provisions for CFS shear walls. The provisions in AISI S213 [1] are capacity based and only provide tabulated nominal shear strength values for specific and limited wall configurations. AISI S213 lists capacities only for two steel thicknesses of 0.457 mm and 0.686 mm. Other limitations also include the wall aspect ratio, fastener spacing, as well as the framing member thickness. These values are based on full-scale shear wall tests conducted by Serrette et al. [2–4]. Other international codes do not even provide the designers with similarly limited values. Designers in Australia for example have no possibility of designing steel sheathed walls unless they refer back to AISI 213.

Another limitation arises when it comes to earthquake resistance design where an indication of the value of response modification factor, *R*, is required for the design based on the equivalent static method. There is no harmony in the values of *R* prescribed by different international codes. Canadian specification provided in AISI S213 [1] does not prescribe a value for steel sheathed shear walls specifically. A plausible value sometimes used by the designers is the one suggested for wood based structural panel shear walls (4.25 alone or 2.55 in combinations with gypsum board). Following the code literally, one has to choose the value of 1, given by the code for "Other Cold Formed Steel Seismic Force Resisting Systems not listed". For use in the

* Corresponding author. Tel.: +61 7 3365 9117; fax: +61 7 3365 4599.
E-mail address: h.ronagh@uq.edu.au (H.R. Ronagh).

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0263-8231/

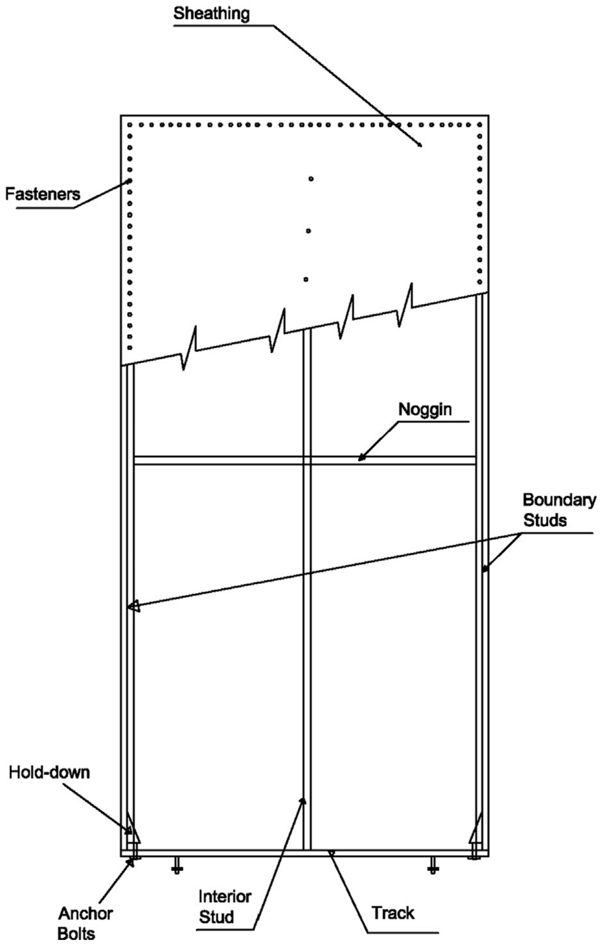

Fig. 1. Typical CFS shear wall with thin steel sheet.

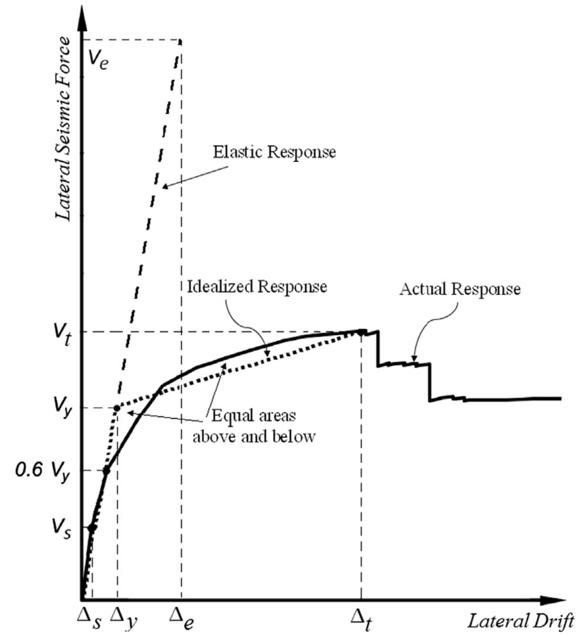

Fig. 2. General structural response, illustrating FEMA's concepts.

United States, the lateral design standard [1] does not enforce any special rule, other than specifications and general provisions for shear walls when the response modification factor is considered to be smaller than 3 in the design. However, for a response modification factor greater than 3, some additional requirements apply, mainly described for diagonal strap bracing members and the anchorage of braced wall segments that resist uplift as well as perimeter members at opening. The alternative between $R \le 3$, with no special requirements, or taking the advantages of $R > 3$, in addition to some essential detailing, is permitted only for the seismic design categories A–C. In the seismic design categories D–F, the designer must use the special seismic requirements to ensure that the system behaves properly in high seismic regions even though an R equal to or less than 3 is used. When read in conjunction with ASCE/SEI 7 [5], these translate to $R = 6.5$ for steel plated CFS walls with special detailing. Finally, the Australia/New Zealand cold-formed steel structures standard, AS/NZS4600 [6], requires that when cold-formed steel members are used as the primary earthquake resisting element, the selected response modification factor shall not be greater than 2, unless specified otherwise in the earthquake loading standard of Australia, AS 1170.4 [7]. Also, all structures shall be designed for the actions and combination of actions specified in AS 1170.4.

As explained, steel sheathed CFS walls are preferred options in many instances though there are very few studies on the seismic performance of these systems. On the other hand, the available date on the load carrying capacity of these systems under seismic loading is limited to a couple of thicknesses and a handful of screw configurations. Hence, the aim of the current research is to investigate the behavior of steel sheathed walls more in-depth with a view to find the lateral resistance capacities, as well as failure modes for a wider range. This evaluation is completed by estimation of the seismic response modification factor for all tested panels, followed by a comparison with the recommended values in structural codes for the R factor.

2. Past studies' and standards' review

In an experimental investigation, Serrette [2] conducted both static and cyclic load tests on walls comprising $3-1/2 \times 1-5/8$ studs spaced at 24 in. Double studs (back-to-back) were used at the ends of the walls. Tests included panels with many different types of bracing and steel plate sheathing. The sheathing or bracing was placed on only one side of the panels. The tests were planned to answer remaining questions on OSB and plywood sheathed walls, to obtain design data for panels with high aspect ratios, and to obtain design data for walls with steel X-bracing or steel sheathing. Failure of steel sheathed panels resulted from the rupture of the steel plate along the line of screws at the edges. Diagonal “tension field” patterns were not observed although this was not verified by actual strain gauges. Decreasing the fastener spacing and increasing the steel sheathing thickness was effective in increasing the maximum load. The maximum loads for panels with an aspect ratio of 4:1 were similar (within 10%) to those for OSB panels with the same aspect ratio and fastener spacings. Displacement at maximum load was 2 in. or more for the panels with an aspect ratio of 4:1, and averaged 1.30 in. for the panel with a ratio of 2:1.

Kawai et al. [8] conducted a series of full-scale experimental tests on different CFS lateral bracing systems including steel plates. Of particular interest were the in-plane shear resistances of the specimens as well as their ductility. They concluded that while the strap-braced frame was very ductile with remarkable pinching behavior, the walls with thin steel plates, plywood and gypsum board showed lower ductility and moderate pinching. They also claimed that the behavior of walls with a combination of two different lateral bracing systems was reasonably close to the behavior of the two bracing systems superimposed.